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OPTIMAL FRP STRENGTHENING OF MASONRY ARCHES

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Abstract

When conservation of historical constructions goes beyond simple modifications or minor superficial reparations, an understanding of the structural behaviour is required. In order to assure a better performance of old buildings during its life expectancy, an original representative structural model needs to be found first, and then a different strengthened structure has to be analyzed. Semicircular unstrengthened and strengthened masonry arches were numerically modelled to evaluate the effectiveness of different FRP strengthening proposals. The paper presents results and conclusions regarding the optimal strengthening of masonry arches.

Key Words

FRP, strengthening, masonry arches

1 Introduction

Masonry is a heterogeneous material composed of units jointed by dry or mortar joints. Stones, ashlars, adobes and bricks have been used as units, which can be joined together using mortar or just by simple superposition. With these two components, units and joints, a large number of arrangements can be carried out. Nevertheless, the mechanical behaviour of the different types of masonry exhibits generally a very low tensile strength. This property is so important that it has determined the structural form of historical constructions.

The use of stone lintels to support masonry above openings in walls had not allowed large spanning distances due to the low tensile strength of the stone. Therefore, the change from linear to curved structures, i.e. arches and vaults, represented a significant structural advance, which allowed to replace stone and timber lintels in walls, with stone or brick masonry spanning wider openings. Indeed, in curved elements, it is usual to find only compressive stresses in a given section and consequently no tensile resistant materials are required.

As part of the widespread European cultural heritage, historical masonry constructions, namely arches and vaults due to their structural significance, deserve particular

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attention because their preservation is a current issue since the majority of them are of considerable architectural and historical importance.
Ageing effects, movements in the abutments or other accidental factors (like ground motion) can cause structural damage on elements belonging to arches, thus affecting their global stability. In order to appraise maximum resistant loads, deformation patterns and collapse mechanisms of masonry arches, a good understanding of their structural behaviour is required. Because structural remedial measures might be needed after a structural evaluation, a significant concern in actual research is the need for efficient strengthening techniques to re-establish the performance of these structures, preventing its brittle collapse when subjected to ultimate state limits.
Among the innovating techniques to rehabilitate deteriorated structures, there has been an increasing interest in composite fibre materials, commonly known as fibre reinforced polymers (FRP). These materials present several advantages, as low specific weight, corrosion immunity and high tensile strength. Their flexibility and somewhat easy application allow a wide range of intervention scenarios.
The use of FRP in special applications in construction is highly attractive and cost-effective due to durability improvement, reduced life-cycle maintenance costs and also savings from easier transportation and enhancement on site-productivity, Triantafillou and Fardis (1997). However precautions using this material must be taken, due to its brittle behaviour.
In order to evaluate the effectiveness of masonry vaults strengthened with FRP, a combined experimental and numerical research project has recently been started at University of Minho. This paper presents numerical analyses of non-strengthened and strengthened semicircular masonry arches, in order to evaluate the effectiveness of different carbon FRP (CFRP) strengthening proposals. The arches are loaded at the quarter span, including their self-weight, being the nonlinear behaviour of the materials considered in the analysis. Indications about width and optimal length reinforcement are established based on the results obtained.

2 Analyses of semicircular masonry arches

For semicircular masonry arches without reinforcement under the application of a concentrated load at the quarter span, four plastic hinges are expected to occur, Heyman (1982). Since the mechanisms related to failure of masonry structures strengthened by FRP sheets or laminates are not yet well known, attention should be given to this matter and experimental research should be focused on the structural behaviour of reinforced arches with FRP.
In order to choose the most favourable strengthening option, namely the optimal width, length and location of CFRP strips, numerical analyses on semicircular arches strengthened with CFRP sheets were carried out. Full and partial length reinforcement with CFRP strips was considered (at the intrados or at the extrados). Additionally, partial length strengthening was considered simultaneously at the intrados and at the extrados.

2.1 Constitutive model for masonry

The approach followed here, concerning modelling, was based on the micro-modelling strategy, where the units behave in a linear fashion and the damage is concentrated in the relative weak masonry joints. Therefore, a composite interface model, formulated within the framework of plasticity, which includes tension cut-off for mode I failure, Coulomb friction envelope for mode II failure and cap mode for compressive failure was considered to model the nonlinear behaviour of the joints, Lourenço and Rots (1997). Due to the low dilatancy exhibited by masonry joints, the model was formulated in the context of non-associated plasticity.

Previous uses of this multisurface constitutive model to analyse strengthened masonry arches allowed to obtain satisfactory results, in terms of load-displacement curves and failure mechanisms, including peeling of CFRP sheets from the masonry, Lourenço and Martins (2001).

2.2 Geometry and mechanical properties of the masonry arches

The semicircular masonry arches have a 150 cm radius, 90 cm width and 10 cm of ring thickness. The displacements at the abutments were restrained in both orthogonal directions, see figure 1a.

Numerical analyses were performed with a finite element program including discontinuities by means of interfaces. The mesh adopted in the analysis includes eight-noded plane stress elements to represent the masonry units, six-noded interface elements to simulate the unit-mortar and masonry-CFRP joints and three-noded cable elements to represent the CFRP, figure 1b. Here, it is noted that zero thickness interface is assumed for the joints. Each masonry unit was modelled with four elements. A monotonic incremental load was applied at the quarter span. The self-weight of the arch was also included in the analysis.

The properties used to simulate the unit-mortar and masonry-CFRP interfaces were obtained from mean values of previous works, see Valluzzi et al (2001) and Lourenço and Martins (2001). Here, the following values were adopted: i) for masonry, a Young's modulus of 950; for the masonry joints, a tensile strength of 0.2 and a shear strength of 0.3; ii) for the composite, a shear bond strength of 2.5, a tensile bond strength of 0.44, a Young's modulus of 230000 and a tensile strength of 3430 (all in N/mm^2). Other values adopted in this paper include: for all joints, a friction angle with 0.75 tangent and zero dilatancy. The softening properties were estimated. Phenomena that can influence the mechanism of failure, like the distance between the strips and like their width, are not considered in the model. Especially the latter phenomenon can lead to undesirable brittle failure behaviour, Tommor and Melbourne (2003).

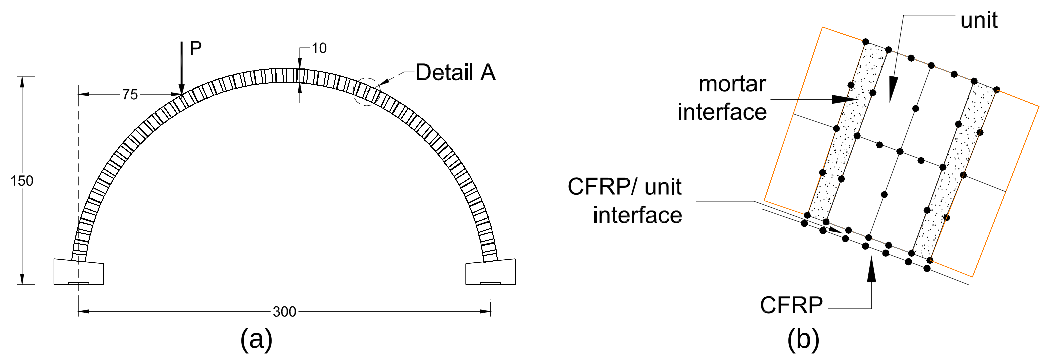


Figure 1 Semicircular arches: (a) Geometry [cm]; (b) Detail A: modelling of the masonry arch and CFRP strips.

2.3 Unreinforced masonry arch

The analysis of the unreinforced arch was carried out in order to identify the maximum load capacity and the mechanism of collapse. This first analysis gives a general overview on how the structure behaves under the loading pattern. The deformed configuration provides an indication on its behaviour and where subsequent arches should be reinforced.

Both the displacement of the node beneath the load application and the incremental deformed shape at the peak load are shown in figure 2. The figure shows that interface opening produces separation between the bricks, located below the loading point. Failure is characterized by brittle behaviour.

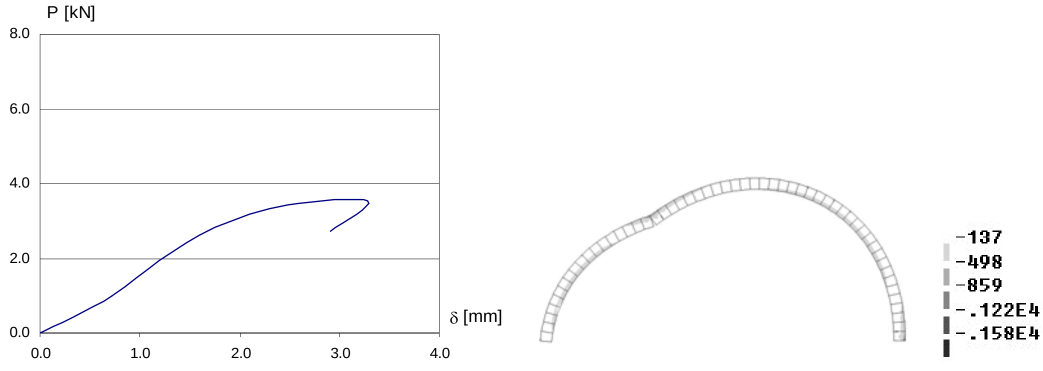


Figure 2 Outline behaviour of the unreinforced arch: (a) Load-displacement curve for the loaded point; (b) Principal tensile stresses [kN/m^2] depicted on the incremental deformed shape (peak load, just before collapse).

3 Masonry arches reinforced with CFRP

After observing the behaviour of the unreinforced arch, a few hypotheses for the arch's reinforcement were adopted. First, full length reinforcement strips placed separately at the intrados or at the extrados were used. Afterwards, symmetrically partial length reinforcement strips were placed at the extrados or at the intrados, separately, and, finally, partial length reinforcement strips were placed simultaneously at the extrados and at the intrados. CFRP strip widths of 6, 14, 18 and 20 cm were selected in order to define an optimal reinforcement width for each hypothesis. Terminology of full or partial length reinforcement hereinafter will be used to refer to strips of CFRP applied at the outer and at the inner surfaces of the arches.

For each reinforcement hypothesis, results associated to 14 cm CFRP strip width are presented in terms of load-displacement curve beneath the application of load and principal tensile stresses depicted on the incremental deformed shape, computed for the peak load, just before collapse. For each CFRP strip width adopted, the maximum load will be graphically related to the reinforcement location.

3.1 Extrados and intrados reinforcement (full length)

Different mechanisms of collapses are developed for the reinforcement placed at the extrados, failure is located in the masonry joints beneath the applied load, where brick separation develops the biggest plastic hinge rotation. The mechanism of collapse produced by these openings prevents the arch from continuing to carry the load. The collapse of the arch is due to sliding at the abutments. Results related to the full length reinforcement at the extrados and at the intrados are showed in figure 3 and figure 4.

For the case intrados reinforcement, failure is characterized by the opening of the joints, in the opposite surface when compared with the extrados, figure 4b.

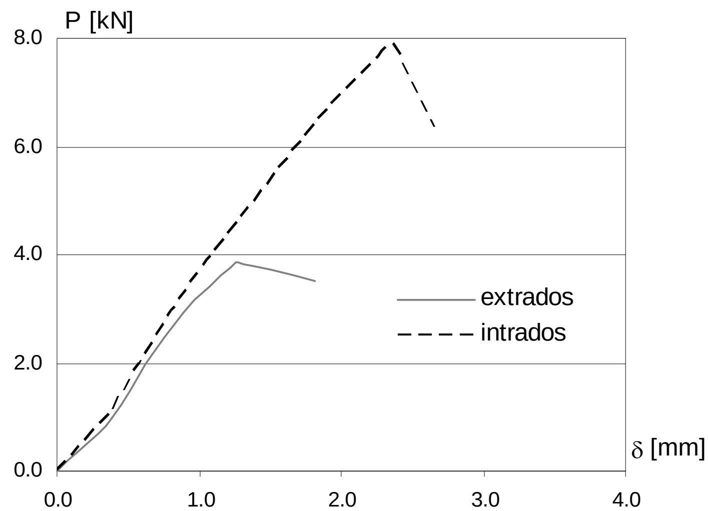


Figure 3 Vertical load-displacement diagram for full length reinforcement (width = 14 cm) at the extrados or at the intrados.

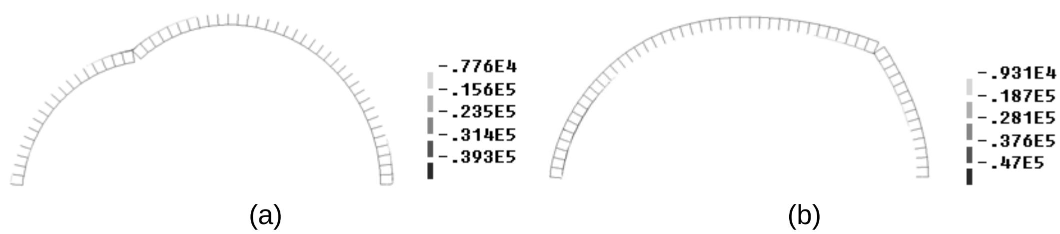


Figure 4 Principal tensile stresses [kN/m^2] for the full length reinforcement: (a) At the extrados; (b) At the intrados.

Some issues are particularly relevant with respect to strengthening of masonry arches with composite materials. When the CFRP reinforcement is placed at the intrados, it contributes to holding the bricks together. Also, experimental research has showed that, for arches strengthened at the extrados, sliding along the joints is the prevalent failure mechanism Valluzzi et al (2001). A solution to avoid such brittle failure can be resolved by optimizing the quantity of applied CFRP, increasing the amount of material near the abutments, where wider and better anchorage is needed. Valluzzi et al (2001) report that an irregular distribution of stresses in the restricted zone located under the CFRP reinforcement is produced by the combination of the small width of the strips and its high Young's modulus. Probably, such phenomena can contribute to lessen the global resistance.

Figure 5 displays a summary for the full length reinforcement option at the intrados as well as at the extrados. Each point in the figure represents the peak load for CFRP widths of 6, 14, 18 and 20 cm, respectively.

Numerical analyses show that arches with full length CFRP reinforcement placed at the intrados present higher peak load values, when they are compared with full length CFRP reinforcement placed at the extrados, for all strip widths considered. For the reinforcement placed at the extrados, the maximum load increases at a steady (low) rate. On the other hand, for the reinforcement placed at the intrados, after 14 cm width the maximum load carried increases at a lower slope than before 14 cm. The peak value for 18 cm width can be considered as the optimal strip width for the intrados reinforcement option. These results depend obviously on the compressive strength of masonry, because a continuously increasing load carrying capacity cannot be found.

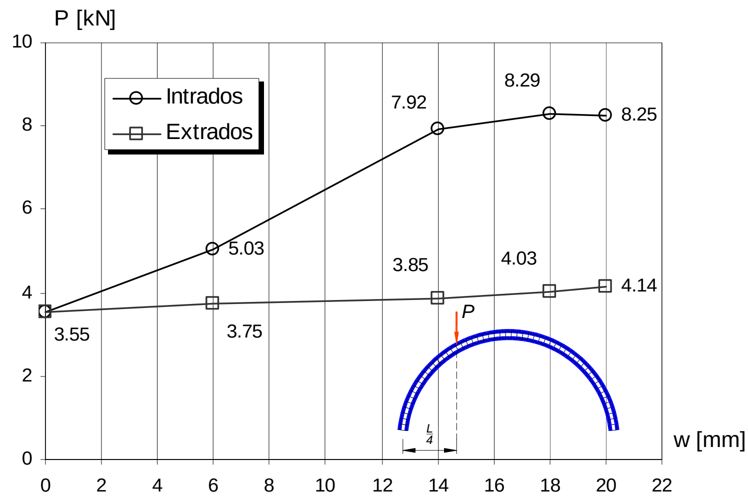


Figure 5 Summary for the full length CFRP reinforcement at the intrados or at the extrados.

3.2 Extrados and intrados reinforcement (partial length)

Based on the mechanism of collapse observed for the unreinforced arch, originated by four plastic hinges, symmetrically partial length reinforcement was adopted, looking for general behaviour enhancement, using less material and placed where it is most required.

Two partial length strips were collocated on the semicircular perimeter of the arch. The configuration adopted was arranged in such a way that two 94 cm length strips of CFRP reinforcement were put symmetrically, at the intrados or at the extrados, centred with the loading point. Using the same numerical model described above, numerical analyses were carried out in order to compute the performance of the arch for the same widths mentioned before.

The structural behaviour considering partial length reinforcement placed at the extrados or at the intrados is illustrated in figure 6. Arches strengthened using partial length reinforcement placed separately at the intrados or at the extrados exhibit similar structural behaviour and collapse mechanisms when compared with the arches strengthened using full length reinforcement at the intrados and at the extrados, respectively, see figure 7. This was expected to occur since the need for reinforcement required by the previous examples was not fulfilled.

A summary of the analyses performed considering partial length reinforcement is presented in figure 8. For both partial length reinforcements, the optimal width is approximately 14 cm. Beyond this value only marginal increase in the load values are observed.

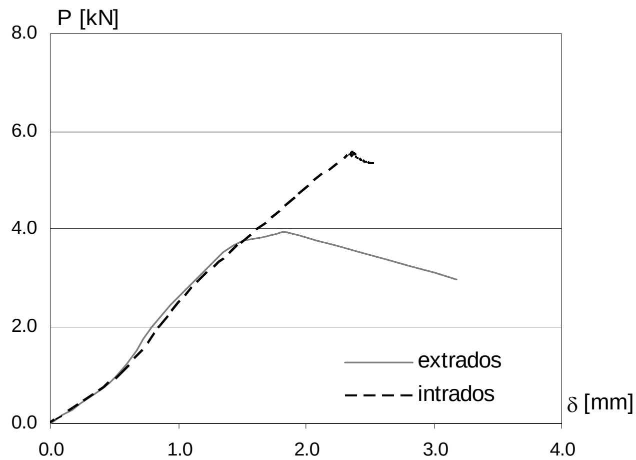


Figure 6 Vertical load-displacement diagram for partial length reinforcement (width = 14 cm) at the extrados or at the intrados.

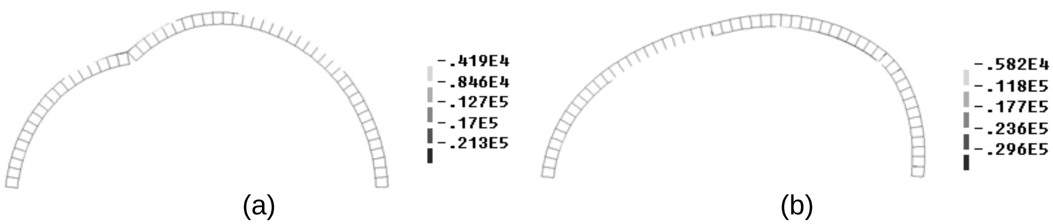


Figure 7 Principal tensile stresses [kN/m^2] for the partial length reinforcement: (a) At the extrados; (b) At the intrados.

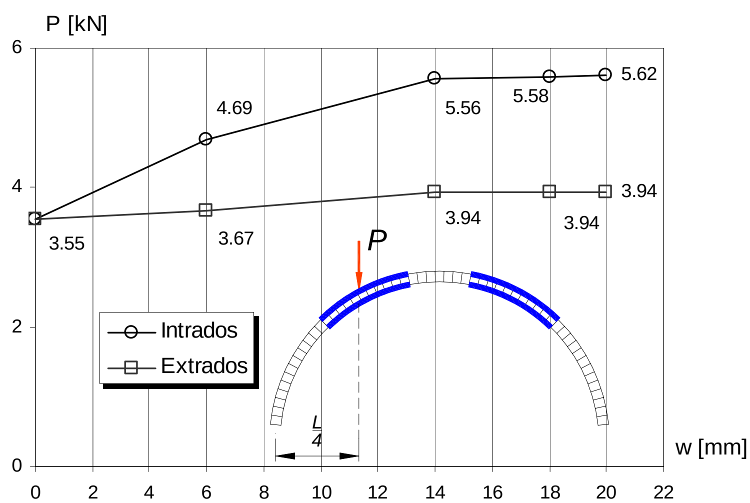
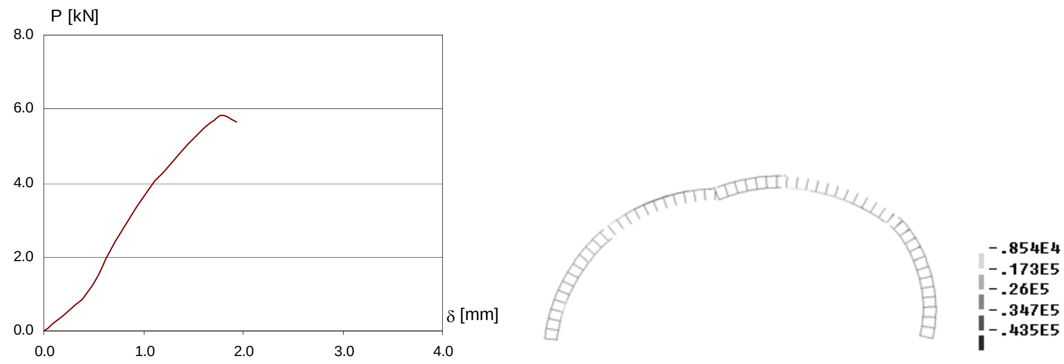


Figure 8 Summary for the partial length CFRP reinforcement at the intrados or at the extrados.

3.3 Simultaneous extrados and intrados reinforcement (partial length)

A combination of the previous arrangement hypotheses was adopted, by placing symmetrically partial length of CFRP reinforcement simultaneously at the extrados and at the intrados. For this hypothesis, higher peak loads were reached when compared with the partial length reinforcement placed only at one surface, as shown in figure 9a. In the mechanism of collapse, the location of the plastic hinges moves towards the zone where reinforcement ends up. A smaller rotation in the plastic hinges was developed which means less damage to the structure. The failure pattern, caused by the sliding between brick and mortar in the first joint, close to the springer, is similar to failure mechanisms as that reported by Creazza et al (2002).



(a) (b)

Figure 9 Simultaneous partial length reinforcement (width = 14 cm) at the intrados and at the extrados: (a) Vertical load-displacement diagram; (b) Principal tensile stresses [kN/m^2].

The summary for simultaneous partial length reinforcement is outlined in figure 10. This figure shows that 6 cm of CFRP width seems to be an adequate amount of reinforcement to reach the highest load sustained by the arch. The same amount of reinforcement (12 cm width) placed only at the extrados gives a smaller peak load and when it is placed just at the inner face of the arch, a quite similar load is reached.

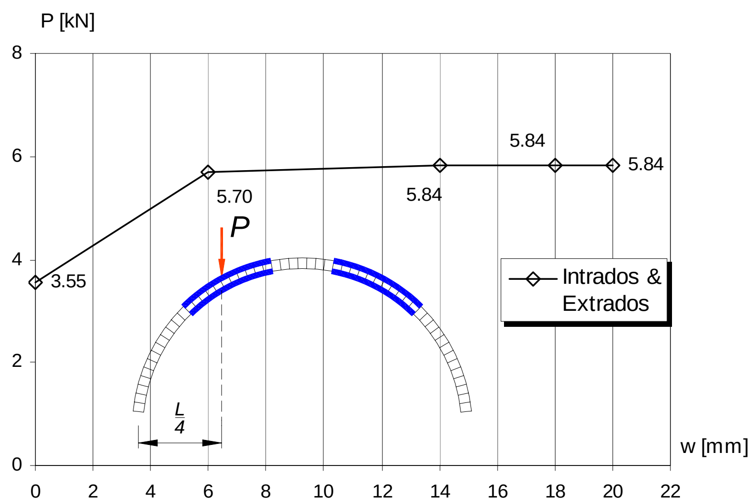


Figure 10 Partial length CFRP reinforcement simultaneously at the intrados and the extrados.

3.4 Additional considerations on the constitutive model

The adopted interface model which represents tensile, shear and compressive failure, in the framework of multi-surface softening plasticity gives very good results when masonry is modelled within the micro modelling strategy, Lourenço and Rots (1997).

One important feature of the adopted interface constitutive model is the coupling of tension and shear softening, because both phenomena are related to the adhesion between unit and mortar. This aspect leads to a zero residual shear stress. However, it is known that the shear stress at interfaces involving FRP reinforcements exhibits a residual value different from zero, CEB-FIP (1990). With respect to this issue, an experimental program is being carried out at the University of Minho in order to characterize the complete shear stress-relative shear displacement at the interface between FRP reinforcement and masonry.

4 Conclusions

Before starting the experimental program on semicircular arches, it became essential to perform previous numerical analyses due to reasons such as corroboration of appropriate equipment available at the laboratory, as well as the optimization of the reinforcement location. The position and the optimal amount of reinforcement need to be found, since beyond certain reinforcement width, insignificant load capacity increments are obtained.

Results have shown that when shift from partial to full length reinforcement placed only at the extrados, no significant increase in load capacity is reached. However, the adoption of full length instead of partial length reinforcement at the intrados originates an important increase in the peak load. All examples studied here have shown that the failure mechanism is highly dependent on the reinforcement option. Careful considerations should be taken since the location of plastic hinges might change, especially when simultaneous partial length reinforcement option is used. For these cases, opening of the joints will tend to develop outside the reinforcement area, causing an undesirable abrupt behaviour.

Considering the optimal CFRP amount for the simultaneous partial length reinforcement, it is possible to compare effectiveness of the same amount of reinforcement among partial selections. The same quantity of reinforcement, placed only at the external surface of the arch presents lower peak load when compared with the intrados reinforcement. For this latter option a similar maximum load is attained when compared with simultaneous partially reinforcement.

For practical matters, the inevitable irregularity of the masonry surface has to be considered, which can lead to poor bond between the masonry and the fibres giving as a result, inadequate strengthening effect.

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